AN ANALYSIS

OF

FOUNDATIONS ON SAND

A THESIS

Presented to

the Faculty of the Graduate Division

Georgia Institute of Technology

In Partial Fulfillment

of the Requirements for the Degree

Master of Science in Civil Engineering

By

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May 1955

ANALYSIS

OF

FOUNDATIONS ON SAND

Approved:

Date approved by Chairman May 24, 1955

ACKNOWLEDGEMENT

The writer wishes to express his appreciation to the many persons whose kind assistance made possible the undertaking of this project.

Especial thanks are extended to Prof. George F. Sowers for his help in selection of the problem and his guidance throughout the investigation. Thanks are also due the other members of the thesis committee, Dr. Harry Saxe and Prof. Richard Poole.

For the necessary equipment and materials used, the writer is indebted to the Law-Barrow-Agee Laboratories, Inc.

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ABSTRACT

This investigation involved a study of the settlement of footings on sand. The primary purpose was to use the elastic properties of the soil in developing an analysis which would closely approximate the settlements of several different footings on sand. The footings ranged from one square foot to four square feet in area and the actual settlements were measured by load tests.

Secondary purposes of the research were to evaluate the effect of inundation on the bearing capacity and settlement as compared with the foundation on dry sand.

The method of approach was to first perform a series of laboratory tests to determine the physical and elastic properties of the sand. A natural river sand having a specific gravity of 2.64 was used. It was sub-angular in shape and uniformly graded with a uniformity coefficient of 1.7. The modulus of elasticity as used was the instantaneous tangent modulus for increasing vertical pressure with constant lateral pressures.

Although it is possible to compute theoretically the stresses at any depth below a loaded foundation, the process is involved and somewhat inaccurate due to the soil not being perfectly homogeneous and elastic. For those reasons, it was decided to make simplifying assumptions with regard to the stress distribution caused by the applied loads. This entailed a division of the affected depth into incremental layer thicknesses and calculation of the average pressure throughout

each layer. Using the relationships between lateral pressure, vertical pressure, modulus of elasticity, plate size and pressure distribution, the theoretical load-settlement curve was established for each footing.

Next, full-scale load tests on the soil were made with the five plate sizes. The tests were performed with the sand in both a dry and a flooded state.

The results of calculations and tests indicate that it is possible to predict the contact settlement at least to an extent which would indicate whether or not excessive differential settlement would occur between two or more footings of the same building. For the dry footings, the maximum error was approximately 50% for the one square foot plate, which had a total settlement of only 0.05 inches at 1000 psf load. For the larger plates, which had settlements from 0.15 inches to 0.45 inches, the maximum error was 29%. Under the flooded footings, the maximum error was approximately 35% after discounting the obviously incorrect experimental values obtained for the 4 square foot plate.

These results can be obtained by the use of laboratory tests at a considerable saving in both time and money as compared with the only other method used--the load test method. Although positive results are indicated, dependency is placed on an assumption as to the variation of lateral earth pressure from the "at rest" to the failure value and considerable work is needed to ascertain the validity of this assumption.

Based on theoretical methods, the bearing capacity of an inundated foundation is reduced by a ratio of the effective weight of the flooded soil over that of the dry material. This reduction is calculated to be approximately 50% while the test results show the reductatation to be closer to 25%. The difference cannot be explained on the basis of our present theories.

No well-defined ratio was found for settlement after flooding when compared at the same applied pressure although the occurrence of additional settlement is clearly demonstrated.

CHAPTER I

INTRODUCTION

The prime requirement of a foundation is to safely transmit to the ground, the stresses induced into it by the supported load. This implies that not only must the bearing capacity (strength) of the soil be sufficient to withstand the load but, also, differential settlement between two or more footings must be of a magnitude which will not cause structural failure of the building.

Although there are several different schools of thought on the bearing capacity of soils, the primary problem to be dealt with in this paper is settlement. In the cohesive soils, the method of analysis of settlement is very well defined. Test procedures have been devised for determining the amount of settlement which will occur in a particular soil under the loading it will carry. While the cohesive soils are the greatest contributors to settlement, the coarse-grained or non-cohesive soils also may play an important role and, as yet, no method of analysis has been developed which utilizes the results of relatively simple laboratory tests.

The "load test" method is one which is being widely used and, when properly conducted, can be a valuable asset in design. The procedure is to use a model of the footing, which is often a one square foot plate. Having located it at critical positions corresponding to the full-scale footing locations, loads are placed on the plate in increments until a total load is reached which is well above the allowable soil pressure. Under each increment of load, the settlement is accurately measured and recorded in the form of a "load-settlement curve".

The load test is useless unless its results are interpreted in terms of the full-sized footing. For sands, the following expression has been proposed from the work of Terzaghi and Peck (1):

$$\rho(\text{foundation}) = \rho(\text{plate}) \times \frac{\left[b_f\,(b_p+1)\right]^2}{\left[b_p\,(b_f+1)\right]^2} \qquad \text{Eq. 1}$$

where: ρ = settlement

b_f = width of footing

b_p = width of plate

The contact settlement of foundations which are five or more feet wide on sand is approximately three times the settlement of a one foot square test plate with the same soil pressure or approximately two times the settlement of a two foot by two foot test plate.

Although streamlined methods and simplified procedures have been developed (2), the "load test" method for determining contact settlement on sands is an expensive and time consuming approach. It is especially so when the area under consideration is large and embraces soils of varying densities. Under these conditions it would be necessary to determine the settlement which could be expected at each footing location in order to control differential settlement.

While one may argue that in this situation, the plate load test will give not only the settlement values but bearing capacity values in addition and is thusly expedient; the bearing capacity of sand is usually not the controlling factor for large foundations. Rather, the settlement characteristics of the soil predominate and, more often than not, the information gathered from the preliminary test boring records is sufficient to set limits on bearing capacity values.

The main purpose of this thesis is to develop an analysis for predicting contact settlement of foundations on sand. This involves studies of the elastic properties of a sand (as determined from laboratory tests) and correlation of the laboratory results with full-scale load tests. A secondary purpose was to note the effect of inundating the footings on the bearing capacity and settlement.

CHAPTER II

THEORY OF SETTLEMENT

The theory of elasticity has come into common use for determining stresses in a soil mass due to an externally applied load. The assumption is made that the stress distribution is the same as that which would occur in a semi-infinite, homogeneous, isotropic, and elastic medium. Under this set of conditions, strain would be proportional to stress. Although it is not necessary for the medium to be elastic for the theory to be applicable, the ratio between stress and the corresponding strain must remain a constant.

Boussinesq (3) developed expressions for the stress components caused by a perpendicular, point, surface load at points in an idealized elastic mass. These components and their coordinates are shown in Fig. 1. The expression for the component, σ_z, is

$$\sigma_z = \frac{Q}{2\pi} \frac{3z^3}{(r^2+z^2)^{5/2}} = \frac{Q}{z^2} \frac{3/2\pi}{\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}} \qquad \text{Eq. 2}$$

The settlements themselves may be caused by the combined effects of consolidation resulting from vertical stresses and lateral and upward displacement due to shearing stresses. The settlement of a foundation on soils other than very soft clays is due primarily to the vertical component; therefore the analysis is usually based on a study of only this vertical component. Since the stress is proportional to 1) the load, 2) the depth squared, and, 3) a function of the ratio of r/z,

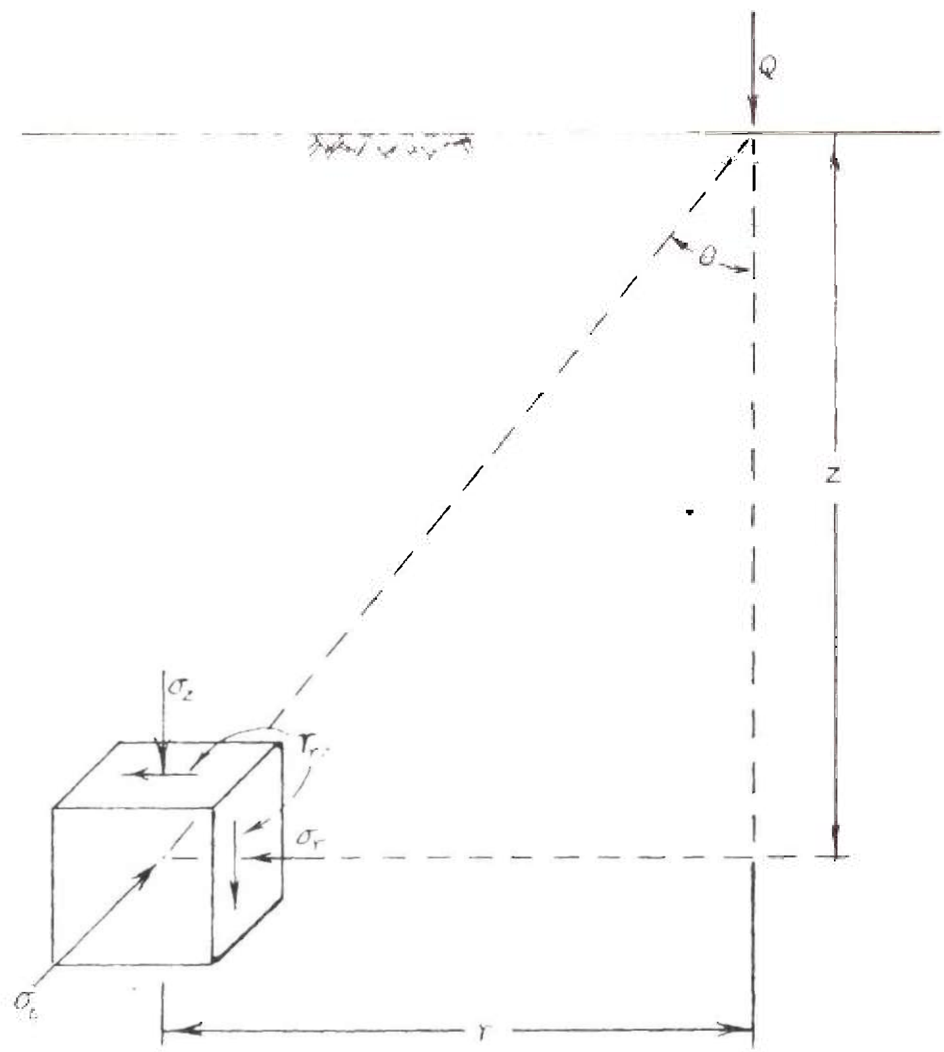


Fig. 1. Stresses in cylindrical coordinates caused by a surface, vertical, point load.

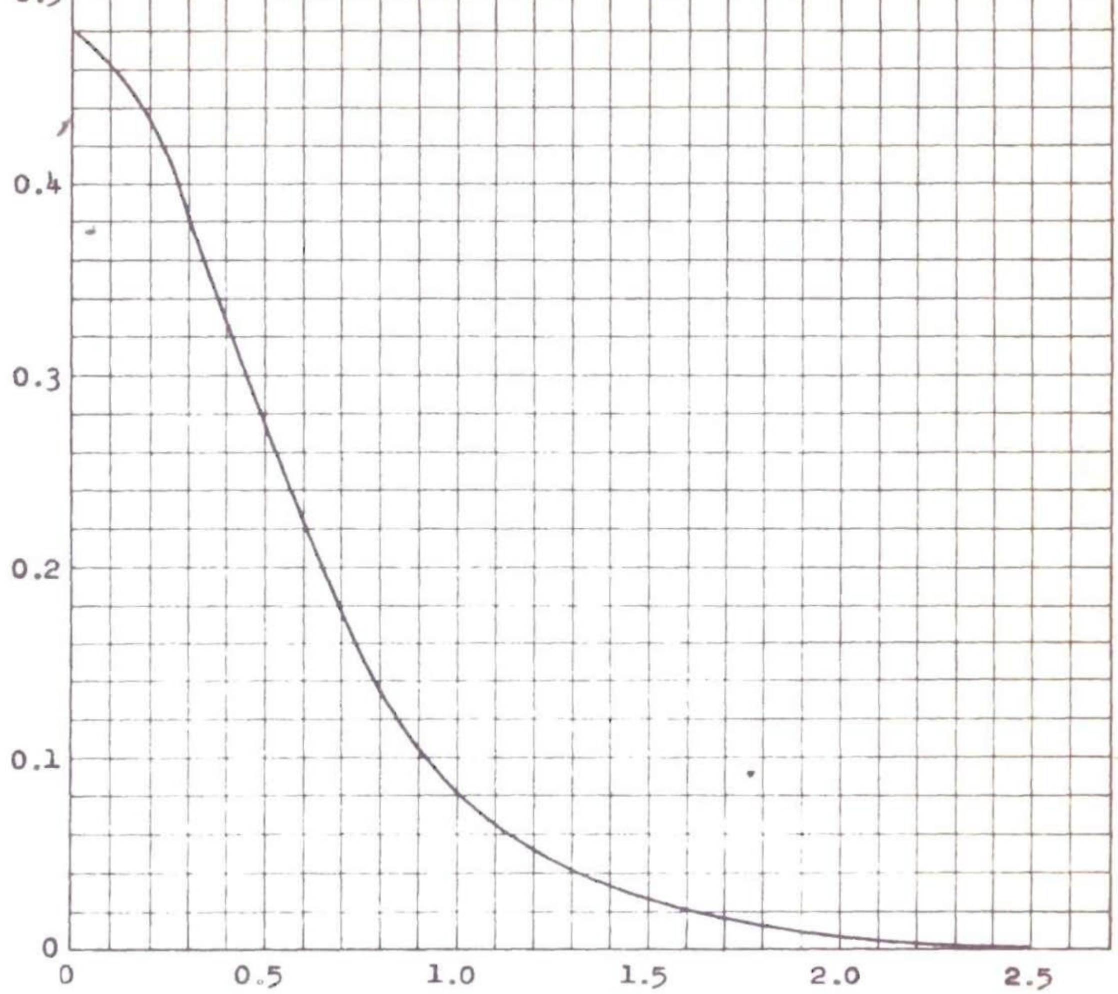


Fig. 2. N_B vs. r/z

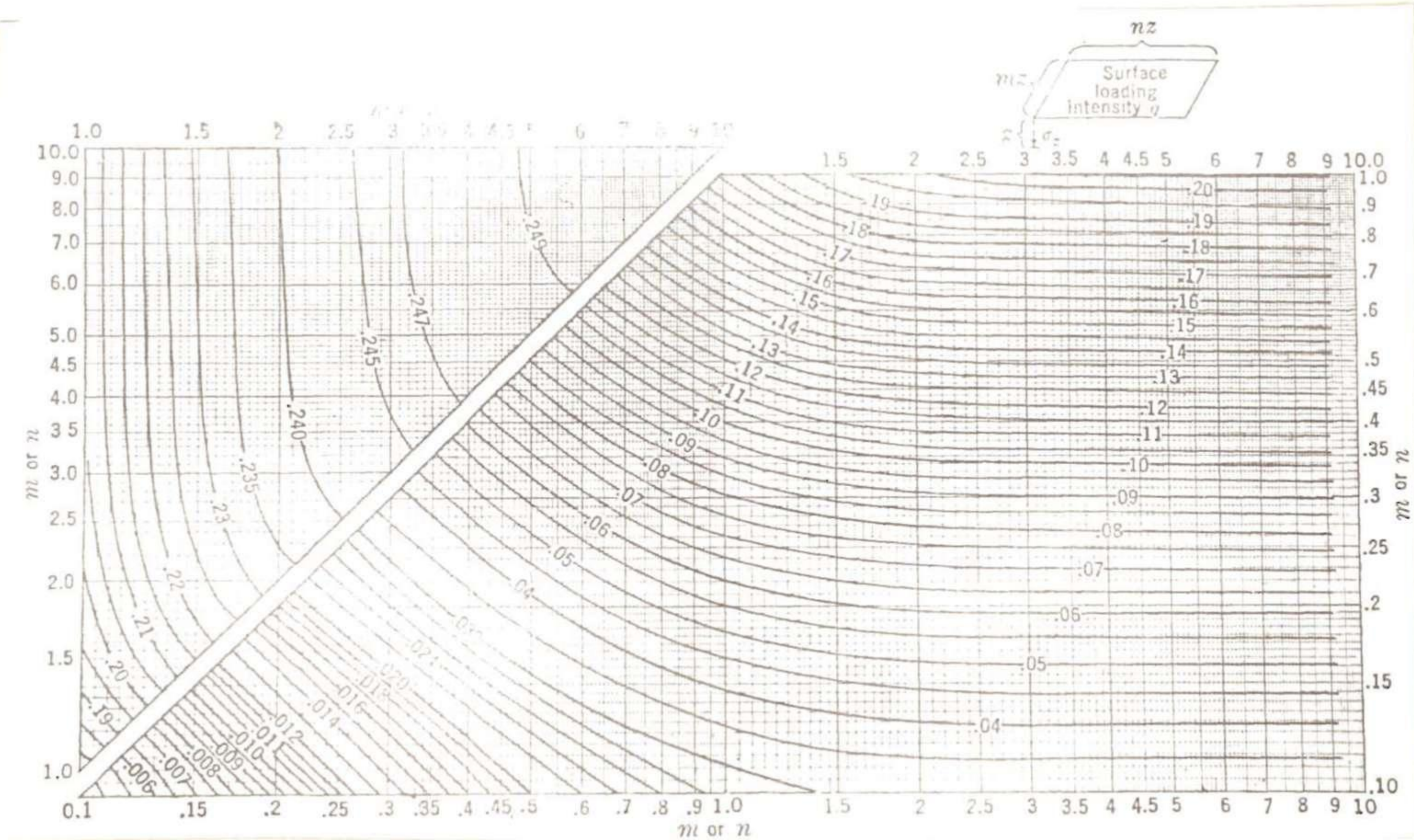


Fig. 3. Chart for determining vertical stresses below corners of loaded rectangular surface areas on elastic, isotropic material.
Chart gives $f_B(m,n)$; $\sigma_z = q \times f_B(m,n)$

the equation can be written as

$$\sigma = \frac{Q}{z^2} N_B \qquad \text{Eq. 3}$$

where: N_B is the Boussinesq index for the vertical stress. A curve of N_B vs r/z is very useful in computations and is shown in Fig. 2.

Newmark (4) has integrated Equation 3 to derive an expression for stresses at a point below a corner of a uniformly loaded rectangular area. This equation is:

$$\sigma_z = \frac{q}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\cdot\frac{m^2+n^2+2}{m^2+n^2+1} + \sin^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1+m^2n^2}\right] \qquad \text{Eq. 4}$$

where: q = uniform intensity of surface loading on a rectangle of dimensions mz by nz.

The absence of the term "z" in the above expression shows the vertical stress at any depth to be a function of the intensity of loading and the dimensions m and n.

A "Chart" (5) which represents Equation 4 for unit loading intensity and for values of m and n from 0.1 to 10 is given in Fig. 3.

Observations of actual settlements have shown that the Boussinesq equations give values which are too large in most cases. Since the conditions existing in practice are seldom, if ever, exactly comparable to those upon which available formulae are based, certain inaccuracies are introduced. These errors are in addition to those due to the non-elastic nature of the soil which are, themselves, of unknown magnitude.

In dealing with cases such as that of a sand stratum near ground surface, the assumption of a proportionality between stress and strain which does not change with depth is far from correct (6). Furthermore, applications of elastic theory to soil cannot be justified unless the stress changes that are to occur in the given case are to be increases only and unless the shearing stresses are to be small relative to the shearing stresses that would cause failure (7). In sand deposits, the pressures on vertical planes tend to be very much smaller than those occuring on horizontal planes. Due to these reasons, surface-loaded sands are eliminated from the range of application of formulae based solely on the elastic theory.

The factors which affect the settlement of foundations on sand are:

1) footing width

2) load per unit area at the footing base

3) instantaneous modulus of elasticity

and, 4) effective confining pressure.

Furthermore, the instantaneous modulus of elasticity at a given vertical pressure (E_i) is a function of the confining pressure (σ_3), which is, itself, directly related to the footing width, vertical pressure, and depth (z) beneath the base of the footing. With introduction of neutral stress (u) caused by a slowly rising water table, the effective weight of the soil is reduced, thereby reducing the confining pressure.

As can readily be seen, the process of settlement is very complex and most investigators have reached the conclusion that the magnitude of these settlements cannot be predicted on the basis of the results of laboratory tests. This writer has attempted to show that an analysis which embraces the interrelation between these variables can very closely predict the settlement. It should be emphasized that while the method herein presented is considered valid, it is based on an assumption as to the variation of the confining pressure with increasing values of vertical pressures. This assumption is partly borne out by other work (8) but is not considered to be a definite representation until verified by further experiment.

For deep foundations the magnitude of this effect is negligible in comparison with the lateral pressures due to the surcharge; however, this effect is of considerable importance when the foundation is at or very close to ground level.

The calculation of the amount and rate of settlement of a foundation requires a knowledge of the stress distribution through the soil after the structure has been erected.

Besides the determination of the stress distribution based on the Boussinesq equations, simplified methods are often employed in practice which assume that the load is carried down on a 2:1 spread and that the stress distribution is uniform on any given horizontal plane.

The intensity of vertical pressure along any vertical line beneath a distributed load decreases with increasing values of the depth "z" below the surface. Therefore, if the compressible layer is very thick, the vertical pressure in the layer decreases appreciably from the top to the bottom. However, the compression of a thin layer depends merely on the average vertical pressure, which is roughly equal to the vertical pressure at mid-height of the layer. When the compressible layer is relatively thin, the change of pressure with depth can be disregarded, and it may be sufficiently accurate to compute the intensity and distribution of the pressure on a horizontal plane at mid-height of the layer.

Laboratory tests have shown that a sand in an at-rest condition exerts a lateral pressure equal to approximately one-half the vertical pressure. When the vertical pressure on a horizontal plane at ground level is increased until a bearing capacity failure occurs, the corresponding vertical and lateral pressures are given by the formulae (1):

$$\sigma_3 = \frac{\gamma b}{2} \tan^3 (45 + \phi/2) \qquad \text{Eq. 5}$$

$$\sigma_1 = \frac{\gamma b}{2} \tan^5 (45 + \phi/2) \qquad \text{Eq. 6}$$

where: γ = effective weight of the soil

b = footing width

ϕ = angle of internal friction of the soil

If the soil is assumed to be elastic, for ranges of pressure below the failure load, the settlement due to an incremental increase in pressure can be computed from the expression:

where: H = thickness of layer

$\Delta\sigma_1$ = increase of the average vertical stress through the layer

E_i = instantaneous modulus of elasticity

Of the three variables, two can be evaluated. The layer thickness can be taken as a fraction of the total depth affected by the pressure at the footing base. The increase of the average vertical stress through the layer can be computed by the Boussinesq equation or by assuming a definite straight line relationship between stress and depth beneath the footing.

This leaves only the modulus of elasticity, which is a function of the confining pressure. The lateral pressure is directly related to the footing width, as shown by Equation 5. A study of compression diagrams (6) shows the stress-strain relationship to be proportional to the internal, or intrinsic, pressure; therefore, with increasing depths below ground surface, all stress-strain ratios of soils increase.

Since the modulus of elasticity of the soil increases, it is apparent from Equation 7 that for equal layer thicknesses and equal stress increases, the layer which is at a greater depth will have the smaller amount of settlement. To compensate for this phenomena as well as for the fact that the load is spread over a larger area with increasing depth, the analysis used here assumes that each layer thickness acts as an independent unit at stresses below the bearing capacity of the soil.

A step-by-step formulation of the analysis is presented in the Chapter on Experimental Procedure.

CHAPTER III

EQUIPMENT

The equipment consisted primarily of a load test rig, a vacuum shear device and a special mold to measure lateral strains due to vertical pressures. A test pit was also a necessary part of the equipment.

A pit six feet wide, twelve feet in length and five feet in depth was constructed and lined with concrete in order to have a nearly water tight compartment in which plate load tests could be made on dry sand. This pit was also provided with a perforated pipe on the bottom through which water could be introduced or withdrawn. The pit was of such dimensions that boundry effects would be quite small when a two foot by two foot square plate was used.

The load test rig consisted of an "A-frame" which furnished a reaction for an hydraulic ram pushing downward on a test plate. Anchorage was provided by using four earth anchors, two at each end of the "A-frame". Fig. 4 shows the set-up. Deflection of the plate was measured by means of two micrometer dial gages mounted independently of the "A-frame" and plate. Deflections under successive increments of loads were measured to one one-thousandth of an inch.

The vacuum shear device is shown in Fig. 5 and was a standard triaxial shear testing machine.

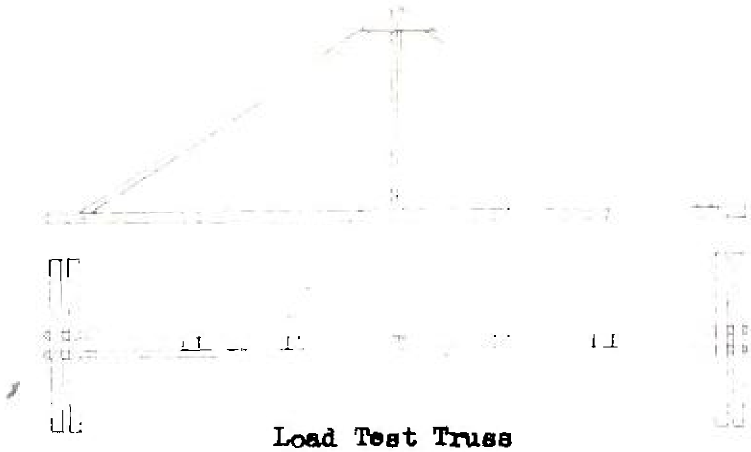
The lateral strain mold was a stainless steel cylinder four inches in diameter (Fig. 6). The cylinder was sawed half way around its circumference so that it consisted of 3/4 inch bands as one half of the cylinder. An SR-4 strain gage was mounted on one of these bands, extending from one end of the band to the other.

The soil used in these tests was a natural river sand from the Chattahoochee River near Atlanta, Georgia. Its physical properties are as follows:

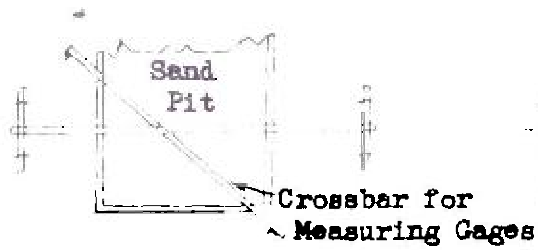
a. sub-angular

b. uniform (see appendix for gradation curve)

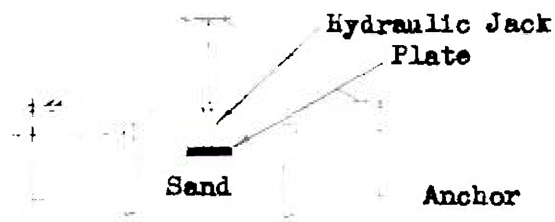
c. specific gravity--2.64.



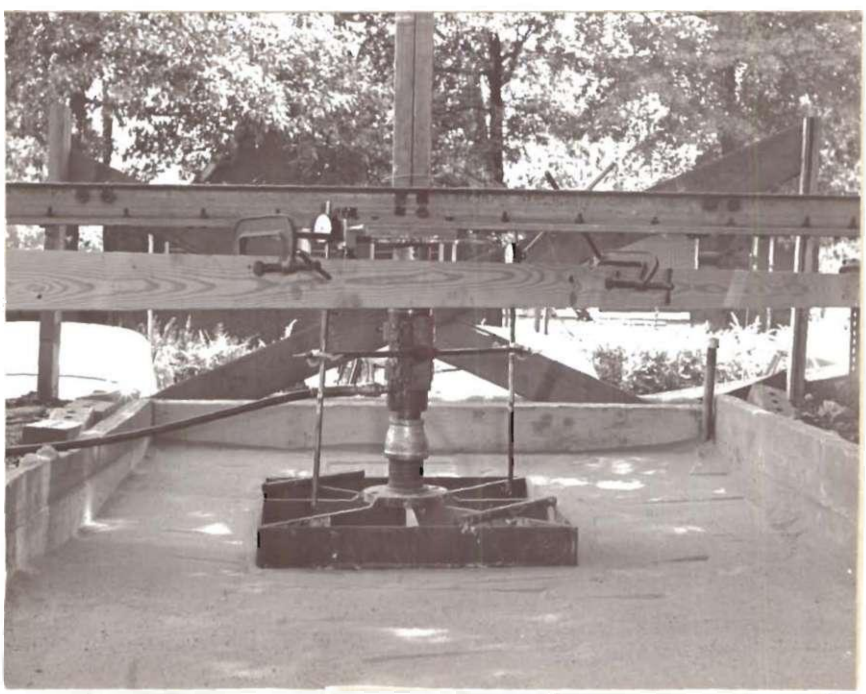
Load Test Truss



Assembly Plan



Assembly Elevation



Four Sq. Ft. Plate in Position

Fig. 4. Plate Load-test Set-up

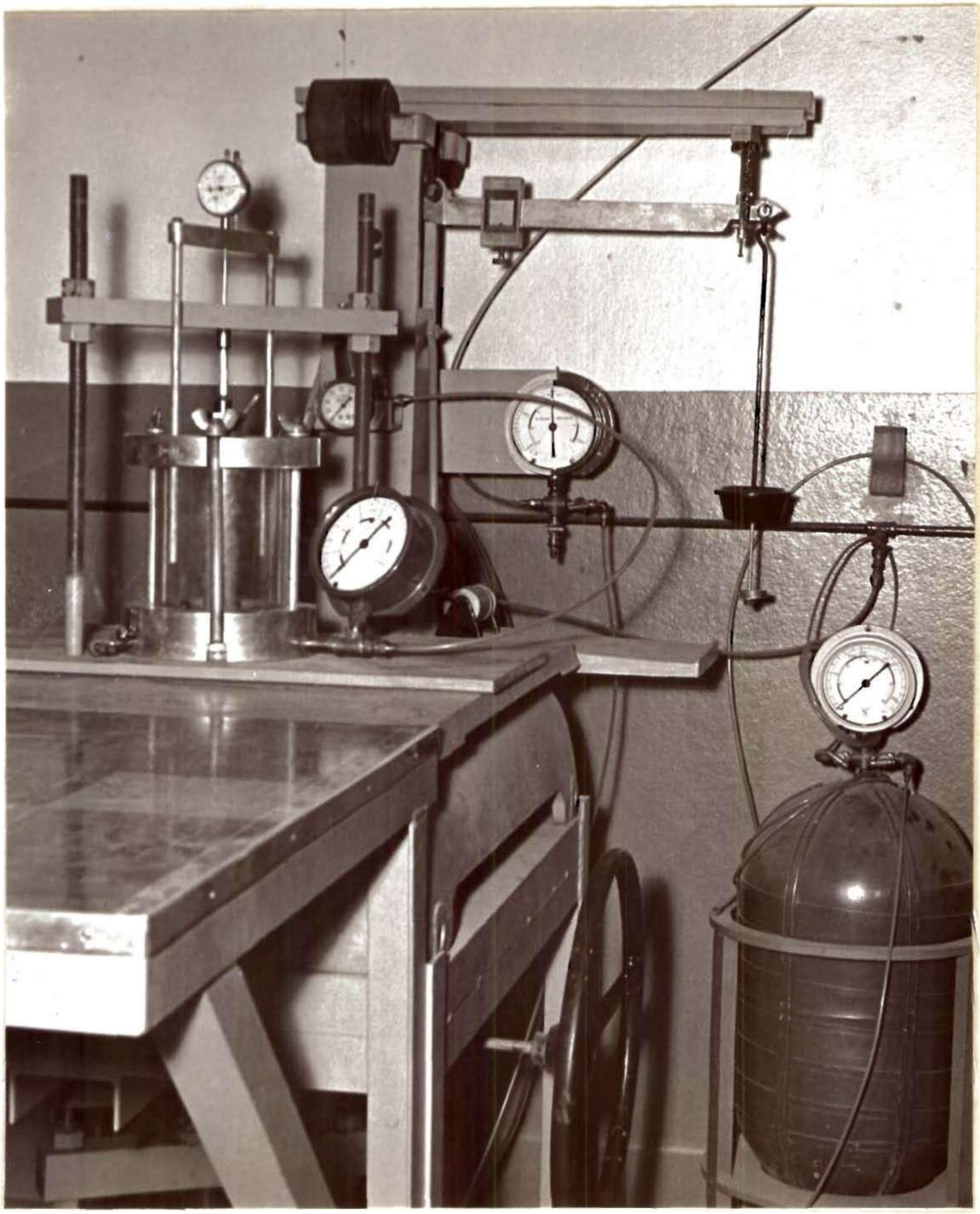


Fig. 5. Triaxial Shear Device

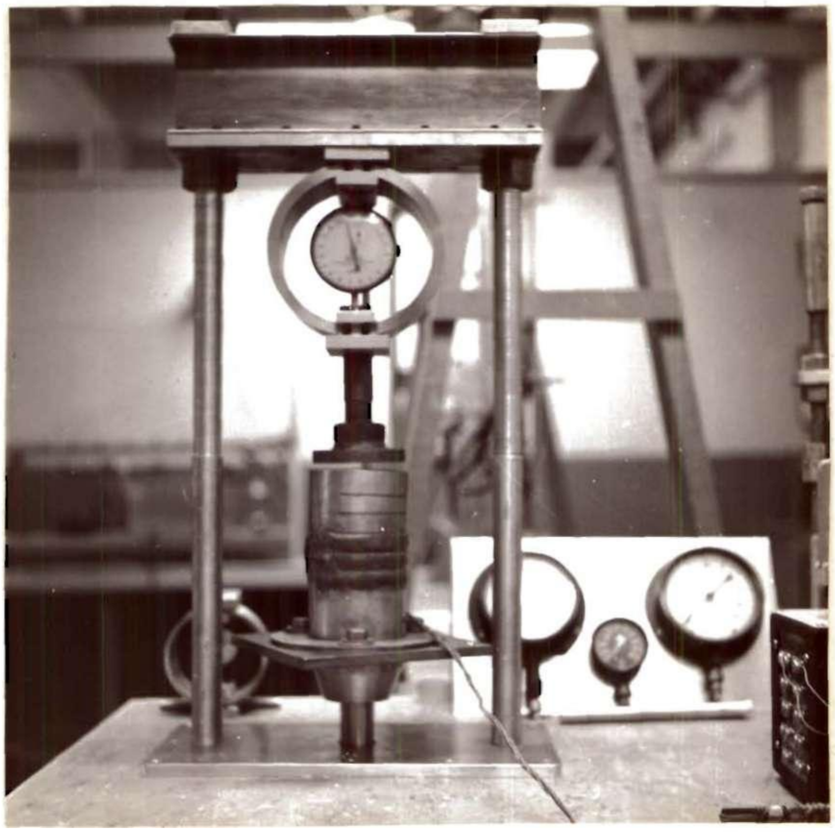


Fig. 6. Lateral Pressure Device

CHAPTER IV

EXPERIMENTAL PROCEDURE

To obtain stress-strain relationships for this sand, vacuum shear tests were performed using values of σ_3, or lateral pressure, ranging from two hundred pounds per square foot to five hundred pounds per foot.

From the stress strain curves, the instantaneous tangent modulus of elasticity for each lateral pressure was determined at various values of vertical pressure. In Fig. 7 these values are shown as modulus of elasticity (arithmetic scale) versus vertical pressure (log scale) for constant values of lateral pressure. This form of curve was chosen to facilitate the determination of modulus of elasticity at low lateral pressures.

Next, the coefficient of earth pressure at rest was determined by use of the special lateral pressure device. The mold was filled with the soil to a point just below the band to which the strain gage was affixed and this material compacted by static pressure under a greater load than would be used in making the lateral pressure measurements. A loose layer was then placed in the mold covering the gage band by approximately one-half inch and a zero reading taken onthe strain indicator. Loads were then applied to the soil and strain readings taken under each increment. These strains were converted into stress measurements and correlated with the vertical pressures. The coefficient thus obtained was 0.49 and used as 0.50 in the calculations.

To use the formula, $\Delta H = \frac{H \Delta \sigma_1}{E_i}$, the variation of σ_3 with σ_1 for the particular soil must be determined. For that purpose, curves of σ_3 versus σ_1 were constructed for different values of "b" (Fig. 8). Equivalent plate widths were calculated for the incremental layer thicknesses as the average of the top and bottom widths. Since the actual variation pattern is now known, these curves were initially assumed to be straight lines connecting the "at rest" values with the "failure" values; the failure stresses assumed to be those given by the Sowers' formulae.

Referring to Fig. 14, it is evident that a logical value of layer thickness should be some ratio of the plate width, this permits the use of a dimensionless parameter which can be extended to any size foundation. Using a stress spread of two vertical to one horizontal, a reasonably close approximation of the theoretical stress spread, a relation was established between width of footing, "b", average pressure through a layer of thickness b/4, width of the area over which the average pressure acted and the effective weight of the soil corresponding to the mid-height of the layer.

From the data thus obtained, the settlement curve for each individual plate was prepared by applying incremental loads to the footing and calculating the resulting settlement. (Figs. 15 & 16).

In order that the method may be clearly understood, the following example is given.

The settlement of the two square foot plate on dry sand due to an increase of footing pressure from 800 psf to 1200 psf is:

From Fig. 14--the average applied stress on the footing is 1000 psf. The average applied stress on an area at the mid-heights of the layers is, (.79) x 1000, (.53) x 1000, (.379) x 1000 and (.284) x 1000 respectively.

Due to the weight of the soil, the total average stress on the layers is increased to 806 psf, 579 psf, 460 psf, and 398 psf respectively.

From the plot of σ_3 vs. σ_1 (Fig. 8), the following values are obtained:

a. Using b' = 19.1"; σ_3 for layer 0-b/4 = 295 psf
b. Using b' = 23.4"; σ_3 for layer b/4-b/2 = 230 psf
c. Using b' = 27.7"; σ_3 for layer b/2-3b/4 = 195 psf
d. Using b' = 31.9"; σ_3 for layer 3b/4-b = 180 psf

From Fig. 7:

a. E for 0-b/4 = $.47 \times 10^5$
b. E for b/4-b/2 = $.68 \times 10^5$
c. E for b/2-3b/4 = $.90 \times 10^5$
d. E for 3b/4-b = $.96 \times 10^5$

The value $\Delta\sigma_1$:

a. Zone 0-b/4 ; $\Delta\sigma_1 = .790(1200 - 800) = 316$ psf
b. Zone b/4-b/2 ; $\Delta\sigma_1 = .530(1200 - 800) = 212$ psf
c. Zone b/2-3b/4; $\Delta\sigma_1 = .379(1200 - 800) = 152$ psf
d. Zone 3b/4-b ; $\Delta\sigma_1 = .284(1200 - 800) = 114$ psf

$\Delta H = \dfrac{(\Delta\sigma_1)H}{E}$ where: H = b/4 = 4.25"

a. Zone 0-b/4 ; $\Delta H = \dfrac{(316)\ 4.25}{.47 \times 10^5} = .0286''$
b. Zone b/4-b/2 ; $\Delta H = \dfrac{(212)\ 4.25}{.68 \times 10^5} = .0132''$
c. Zone b/2-3b/4; $\Delta H = \dfrac{(152)\ 4.25}{.90 \times 10^5} = .0072''$
d. Zone 3b/40b ; $\Delta H = \dfrac{(114)\ 4.25}{.96 \times 10^5} = .0050''$

Total Settlement = .0540"

In the case of the flooded sands, there is a reduction of the effective weight of the soil and since the confining pressure, σ_3, is a function of this pressure, there is an increase in the settlement.

Once again, it was necessary to use a bearing capacity formula, this time one which included the effect of flooding. The Sowers formula, now becomes $q_c = \frac{\gamma' b \tan^5 \alpha}{2}$, resulting in a decrease of strength by a ratio of γ'/γ and necessitating the construction of a new set of curves for horizontal versus vertical pressure. Fig. 8.

To obtain settlement values, the same procedure is followed as is shown in the preceeding example for dry sands with the substitution of the "flooded" pressure-ratio curve for the "dry".

The next step in the experimental work was the determination of the actual settlement of a foundation on sand by means of the plate load tests. Five different size plates were used. These had areas of one square foot, one and one-half square feet, two square feet, three square feet, and four square feet.

In order to achieve a uniform density in the test pit, the dry sand was carefully shoveled in, with a throwing motion which scattered each shovelful rather than letting the entire mass concentrate at one spot. At frequent intervals (approximately each twelve inches of depth) the density was measured by means of the "sand-cone" density apparatus. A very thin walled steel liner was pushed into the soil for a depth of six inches in order to keep the soil from caving-in while digging the hole for the density measurement. A sufficient number of samples were taken from each layer to assure uniformity with the average of the density measurements being 89.9 pounds per cubic foot.

After obtaining the required density, the plate load tests were performed, the soil being removed and replaced after each test. This was done to eliminate any effects of settlement on the density although checks showed negligible amounts of increase due to settlement.

After completion of the tests on the dry sand, an entire series was performed in a flooded condition. The procedure for this series was to first drain the pit (for all but the first plate for which the sand was already in a dry condition), stir the soil and replace it at the correct density. Following this, the plate was put in position and water was slowly introduced through the pipe until the plate was approximately one-half inch below the water surface. The water level was held constant at this elevation and loads were applied to the plate in increments as in a standard test.

The results of these tests (dry and flooded) were plotted as load-settlement curves and are shown in Figs. 15 and 16.

When compared with the measured settlements, the calculated settlements were found to be very much in error. See Fig. 17. In order to correct the differences, new curves of σ_3 vs. σ_1 were plotted which increased more rapidly from the initial conditions and approached a constant before failure. The shape of these curves was arrived at by noting the relationship between the total settlement and the settlement in the uppermost layer as determined for the three square foot plate from the straight-line curves. Working backward from the measured settlements, values of σ_3 were plotted against σ_1 which would produce the modulus of elasticity required for these settlements. It should be borne in mind that the curve was correlated with the settlements of only the three square foot plate and that the good agreement of the other plates is due only to the relation between the various factors which are involved.

CHAPTER V

RESULTS

The load-settlement curves of the plate load tests are shown in Figs. 15 and 16. With the exception of the largest plate, which has a steeper slope and a much smaller bearing capacity than seems reasonable, the "dry" curves exhibit the characteristics commonly attributed to this type of test. As the width of the plate increases, the slope of the settlement curve becomes greater and the load carrying capacity is increased.

Introduction of the neutral stresses caused by the inundation of the plates definitely increased the settlements of all plates except the largest, under which the water apparently decreased the settlements. There is no defined ratio between the settlement on dry sand and that on flooded sand with regard to plate size although the trend is for the percentage of increase to become greater with decreasing footing size at equal pressures.

A study of the bearing capacity of the flooded plates reveals one of the basic weaknesses of the plate load test. The most common way of determining the bearing capacity by a load test is to extend the initially straight portion of the curve. With increasing pressures, the curve gradually becomes steeper until it is once again essentially straight with a steeper slope. A tangent to this latter portion is constructed and made to intersect the extension of the initial straight line portion. This intersection is taken as the bearing capacity.

As can be seen, the latter stage of the flooded curves was generally at lower pressures and steeper than their "dry" counterparts but the combination of increased initial slope and increased final slope gave apparent bearing capacity increases in some cases. This could introduce a false sense of security when interpreting results of plate load tests on inundated foundations which would be subject to possible lowering of the water table.

The fact that the settlement curve of the two square foot plate is less when inundated than when dry indicates the possibility of a "bad" test under the flooded conditions and all results should be studied with that in mind.

The stress-strain curves exhibit no special features or disqualifying characteristics but reveal the manner in which the instantaneous tangent modulus of elasticity is increased for any given vertical pressure as greater values of lateral pressures are employed. The Mohr envelop shows the angle of internal friction to be 36°.

It is evident that one of the most important assumptions in this method is the variation of lateral pressure with vertical pressure. As stated previously, this is an unknown but there is some evidence that the curve as used is to some degree, qualitative if not quantitative.

Tschebotarioff (9) reports some results of cell tests in which σ_3 vs. σ_1 for a compacted sand is as shown in Fig. 18.

The fact that this curve has the same curved shape as the ones used in the calculations herein is very important and a convincing argument for the validity of this method of analysis. The plate load test is essentially a greatly enlarged cell test since the strains are minute in relation to the volume; it is, therefore, logical to conclude that the stress relationships should parallel those obtained in a constant volume triaxial test.

Although the method of assuming stress relationship curves based on the widths of the mid-heights of the incremental layers may be questioned, there is absolutely no question as to the fact that there is more settlement in the uppermost portions of the total layer and a partial progressive failure beginning in the region immediately under the footing. Consequently, the fact that the vertical load spreads horizontally must be accompanied by an increase in the bearing capacity of any deeper horizontal plane since the increased area now becomes the load bearing surface and corresponds to the increase attributed to one footing which is larger than another.

Referring now to Figs. 9 through 13, the results of this investigation are seen as curves of calculated settlement in comparison to those measured by the load tests. The primary objective of this thesis is therefore accomplished since the two curves are in good agreement for the range of pressures up to the elastic limit of the sand. Although there is need for refinement in method in order to calculate curves which are straight lines, this may not be feasible until more is known about the actual variation of lateral pressure with increasing vertical pressures.

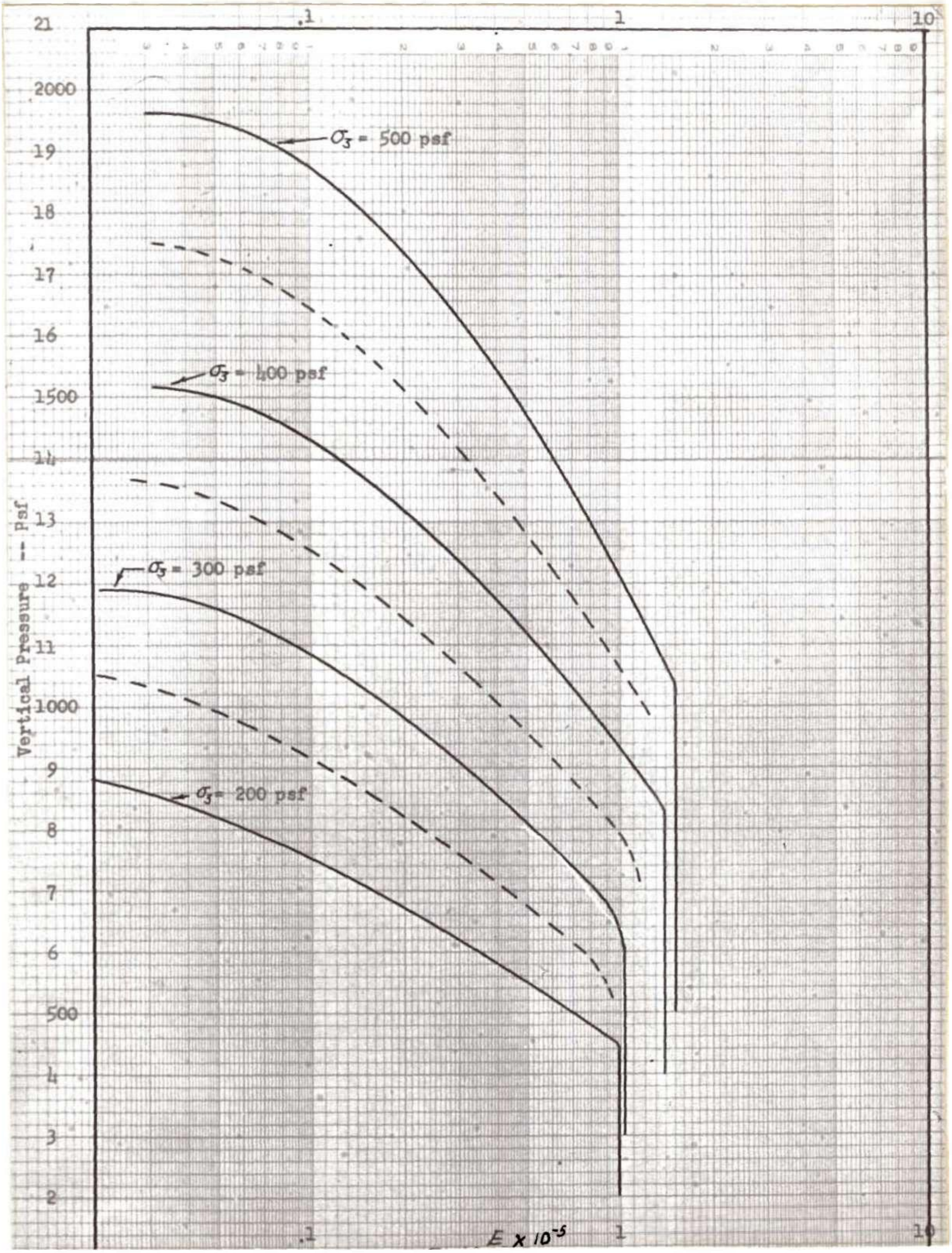


Fig. 7. Instantaneous Modulus of Elasticity vs. Vertical Pressure

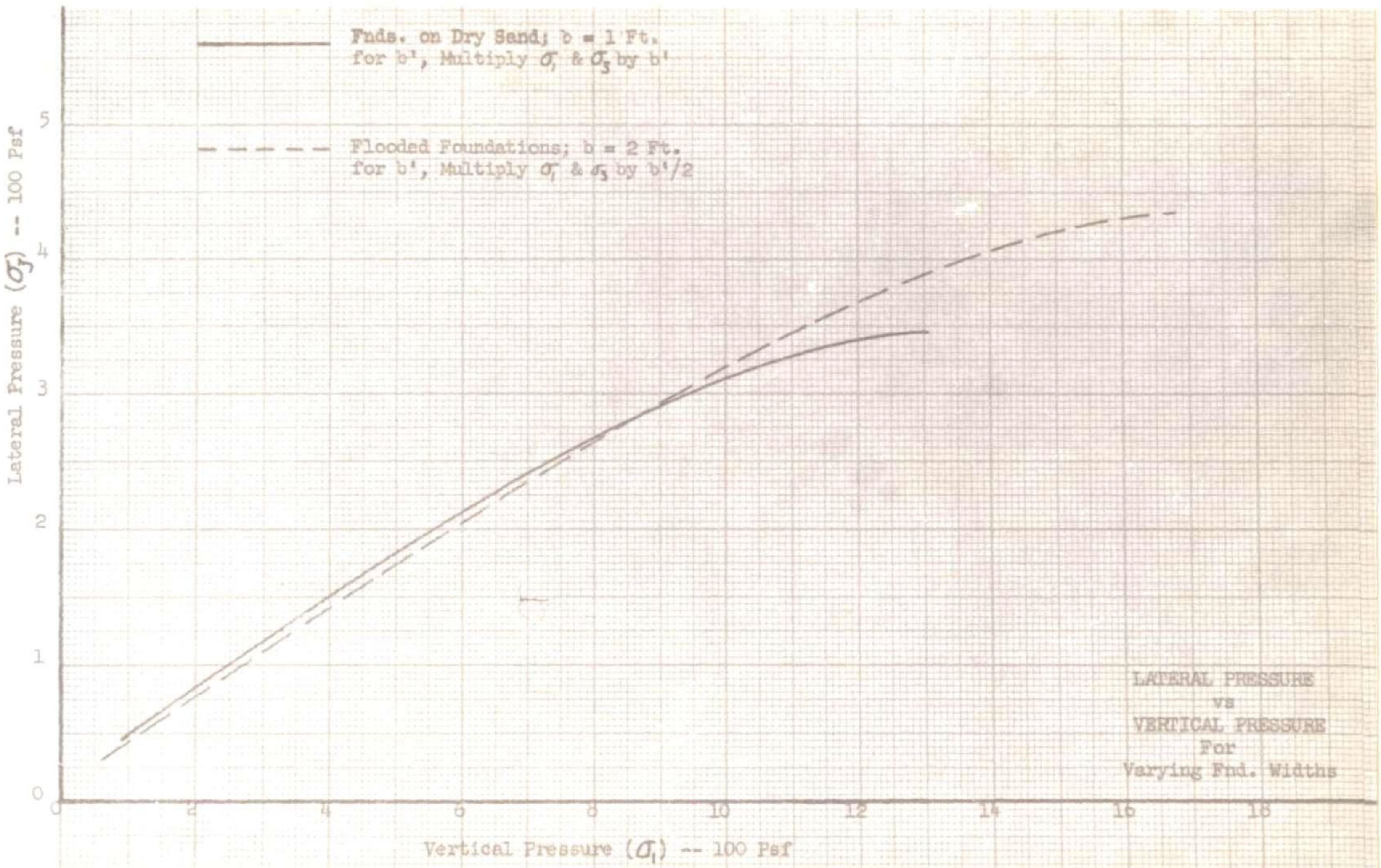


Fig. 8. Lateral Pressure vs. Vertical Pressure for Varying Footing Widths

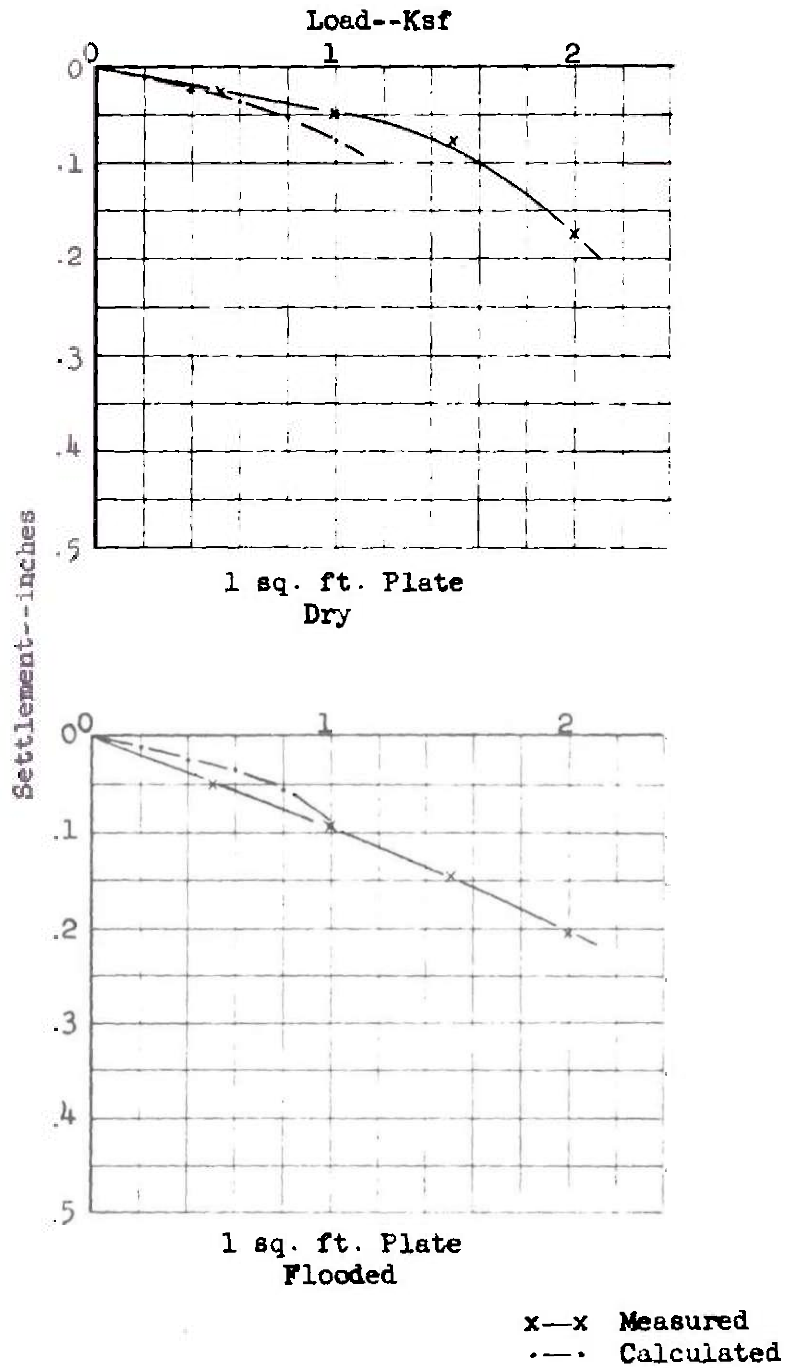


Fig. 9. Calculated Settlement compared to Measured Settlement

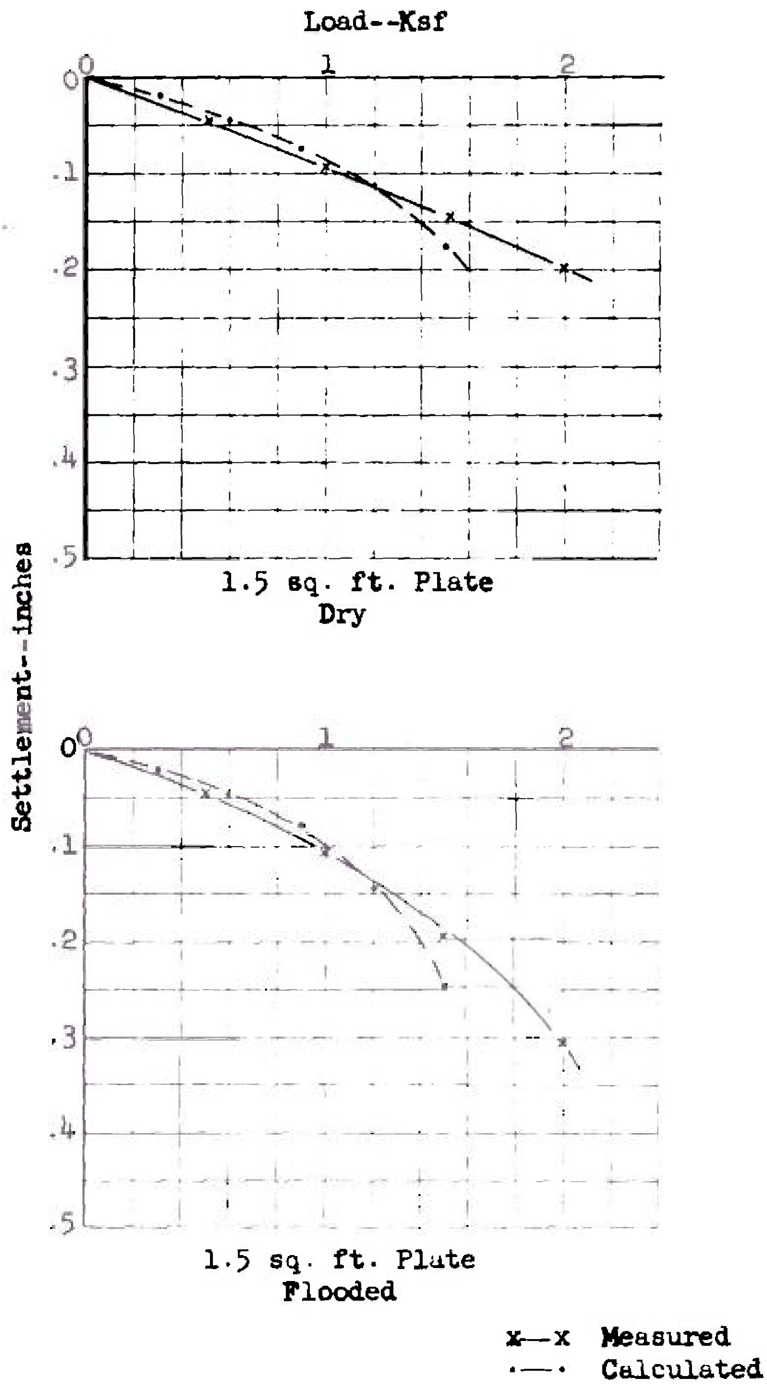


Fig. 10. Calculated Settlement compared to Measured Settlement

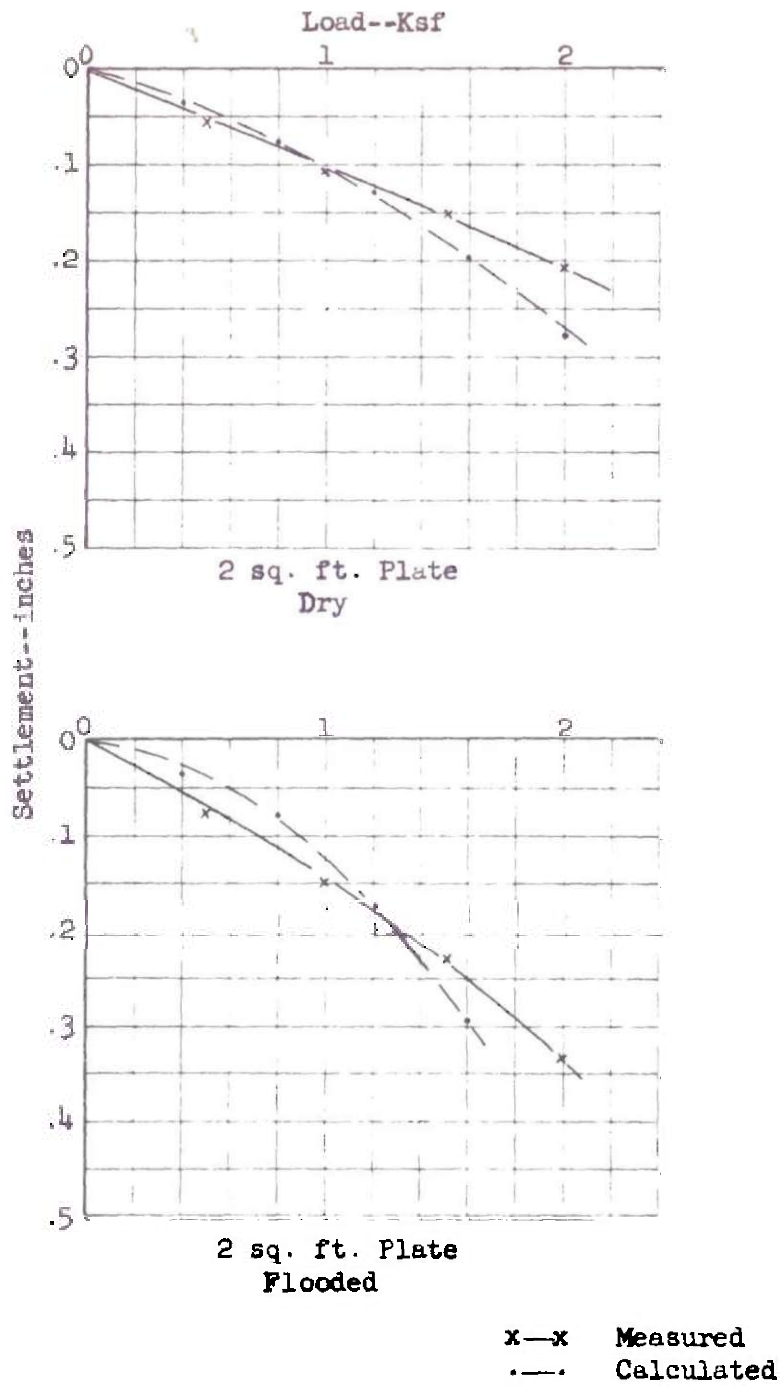


Fig. 11. Calculated Settlement compared to Measured Settlement

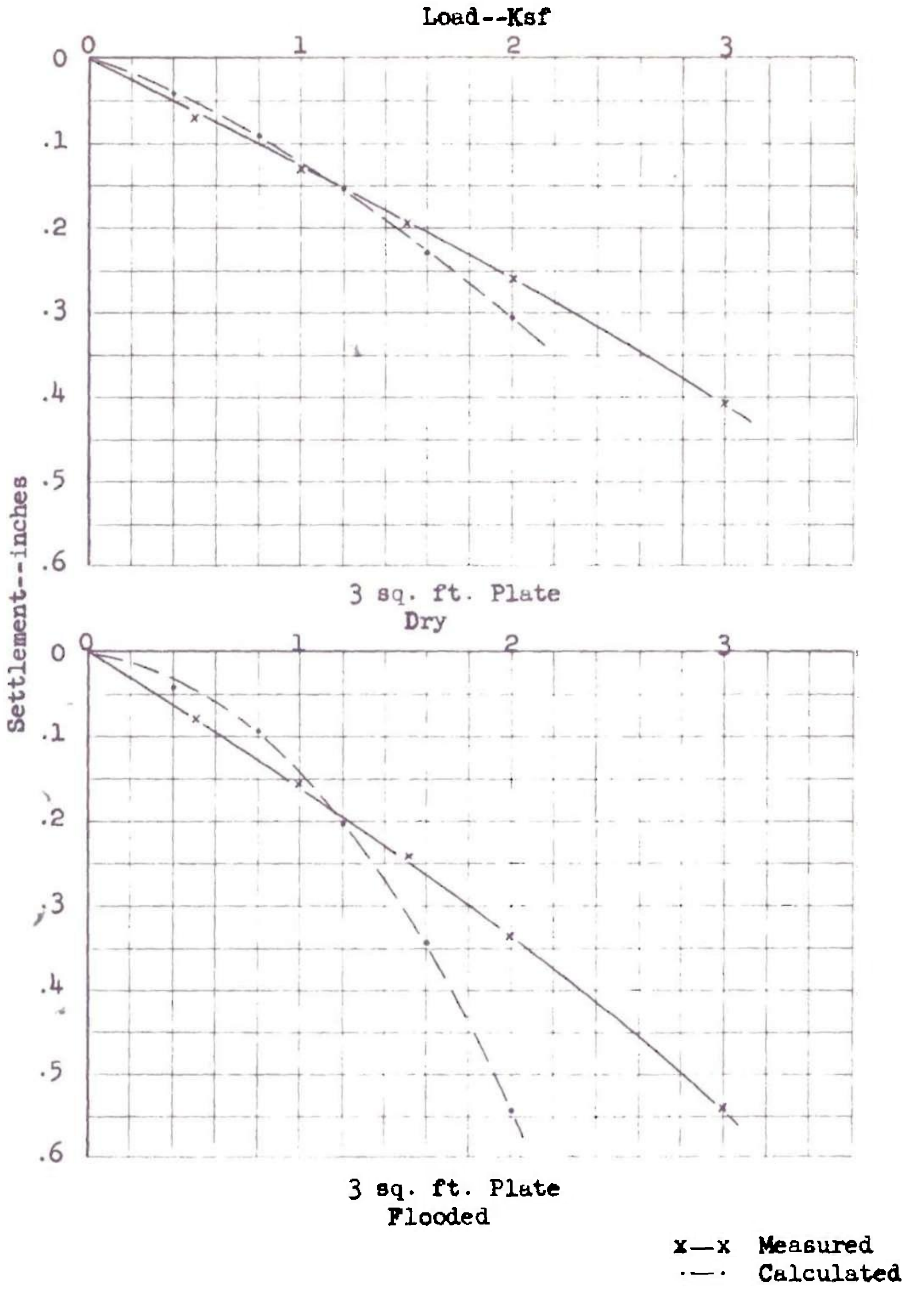


Fig. 12. Calculated Settlement compared to Measured Settlement

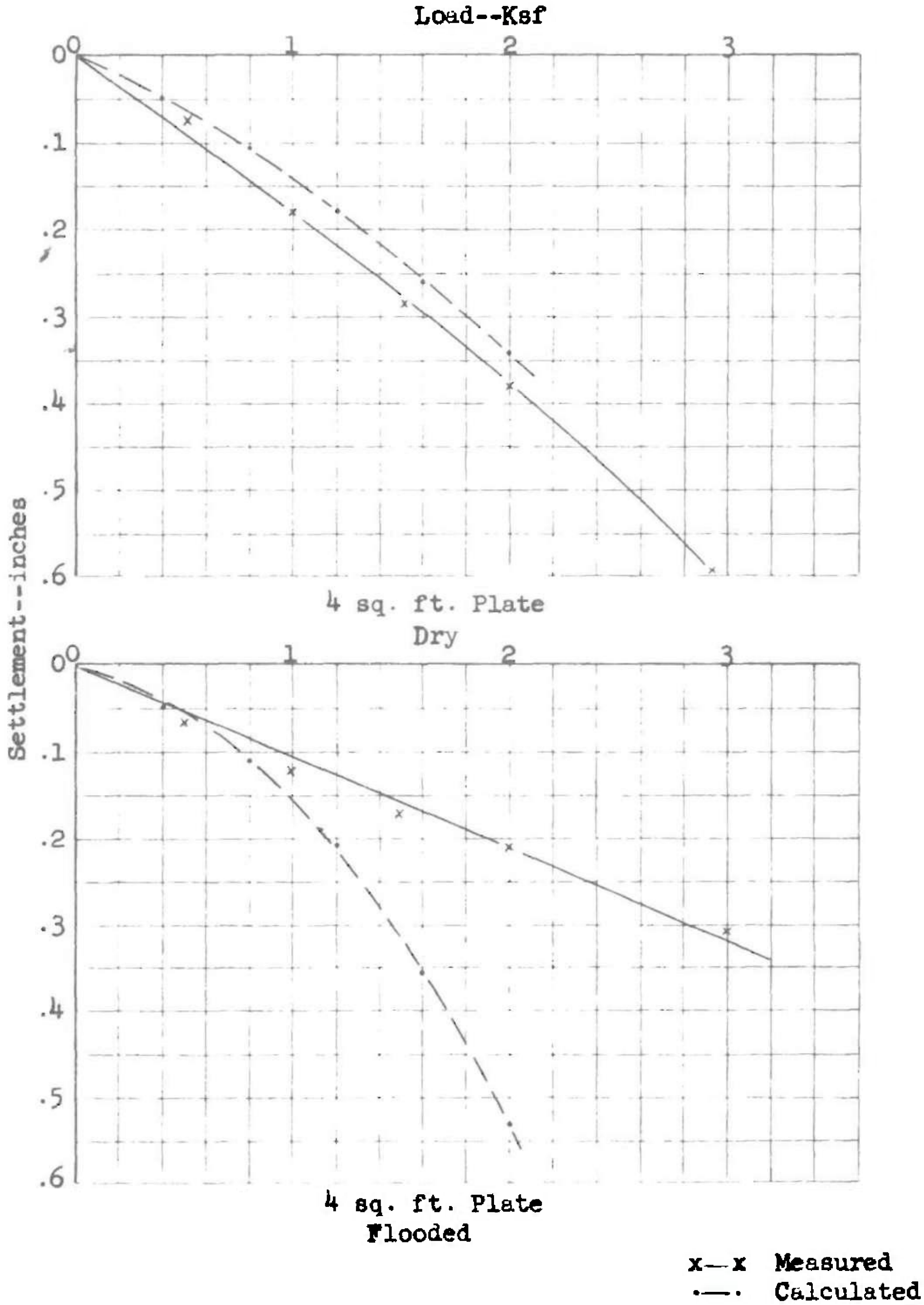


Fig. 13. Calculated Settlement compared to measured Settlement

CHAPTER VI

CONCLUSIONS

The following conclusions have been reached from the results obtained in this work:

1. The bearing capacity and settlement characteristics of dry and flooded sands under loaded footings are functions of the width of the footing. Greater bearing capacity and larger amounts of settlement for equal pressures are obtained as the width of the footing is increased.

2. The bearing capacity and settlement characteristics of sands are influenced by flooding; the bearing capacity is lowered and the settlement is increased when the sand undergoes complete saturation from an initially dry state.

3. The bearing capacity is not reduced by approximately one half due to submergence, as is shown by theory, but to a value somewhere between 50 and 100 per cent of the dry state, probably around 75 per cent.

4. The settlements of footings on dry and flooded sands can be very closely estimated by the use of its elastic properties for the footing sizes used in these tests and probably for others.

CHAPTER VII

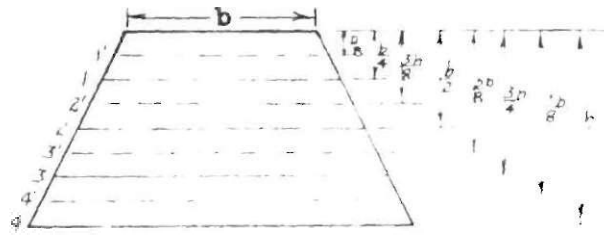
RECOMMENDATIONS

Although it has been shown that it is possible to calculate rather closely the contact settlement of foundations on sand, the variable which has the most importance on the results is the one about which the least is known. Since the method presented could have great import on foundation design it is worthy of further study, especially in the lateral pressure variation. Not only would findings be of value to foundation work but the field of earth pressure in general would benefit.

In this investigation, flooding of the soil was done before any loads had been placed on the footing. Although designs are usually based on this procedure, the correct method would be to base calculations on the effect produced by a rising water table under an initially loaded footing. It is therefore recommended that a program of study be initiated with the purpose of determining just what relation exists between the flooded bearing capacity and settlement as given by loading a footing on flooded sands and the corresponding values as determined by subjecting a loaded footing to a flooded condition.

Additional study is also needed to determine the effect of materials of gradations other than the one used here.

APPENDIX



Loading Factors

Surface	Width	Area	q'	γd (dry)	γd (wet)
1'	1.126b	$1.266b^2$	.790	11.5b	7.2b
2'	1.375b	$1.890b^2$	.530	34.5b	21.7b
3'	1.627b	$2.640b^2$	.379	57.5b	36.1b
4'	1.878b	$3.520b^2$	.284	80.5b	50.6b

Equivalent Widths at Surfaces

b	b'			
	1'	2'	3'	4'
12"	13.5	16.5	19.5	22.5
14.5"	16.3	19.9	22.4	27.2
17"	19.1	23.4	27.7	31.9
21"	23.6	28.9	34.1	39.4
24"	27.0	33.0	39.0	45.1

Fig. 14. Loading Values for Calculating Settlement

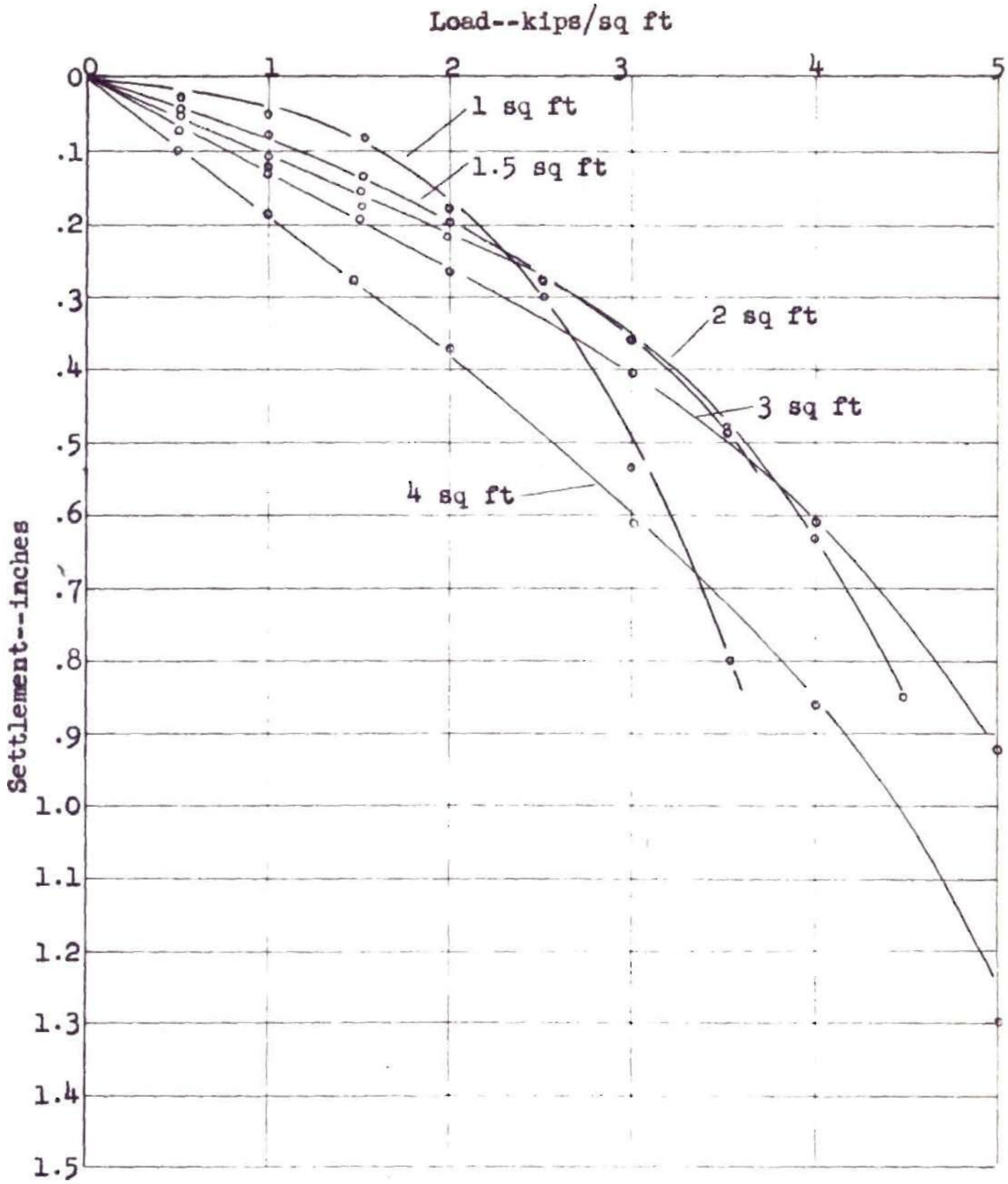


Fig. 15. Load Settlement Curves for Dry Conditions

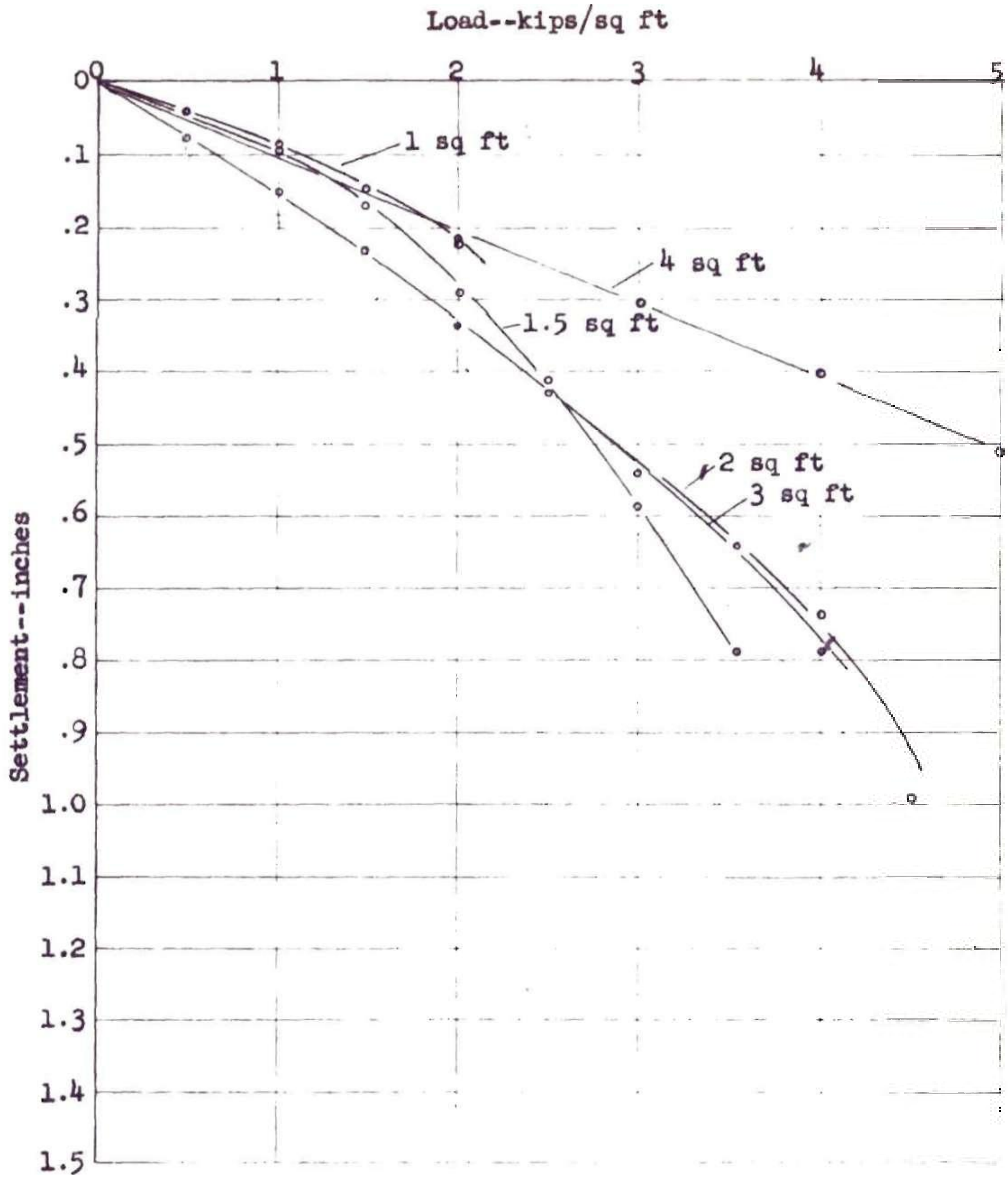


Fig. 16. Load Settlement Curves for Flooded Condition

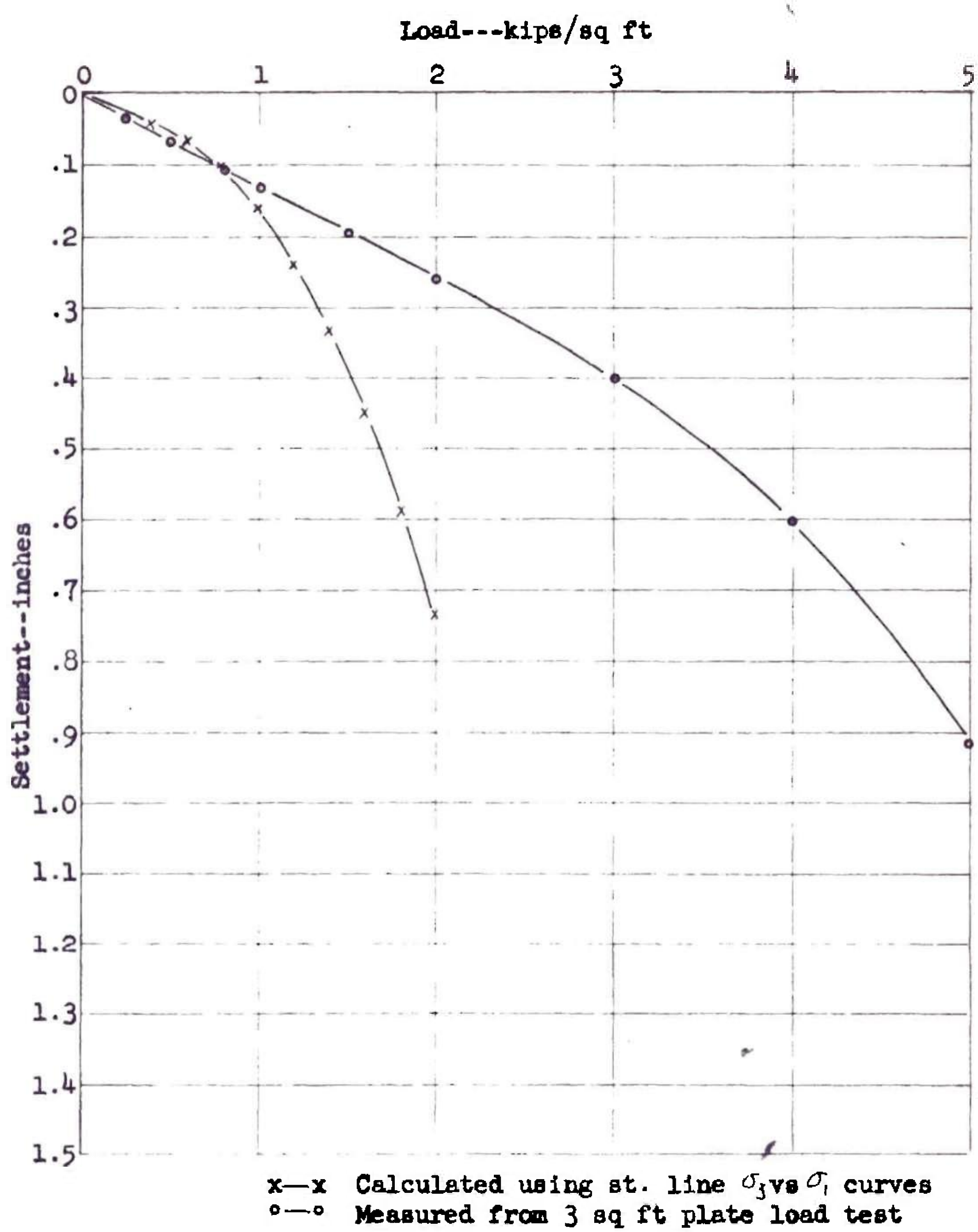


Fig. 17. Load-Settlement Curve
2 sq ft Plate

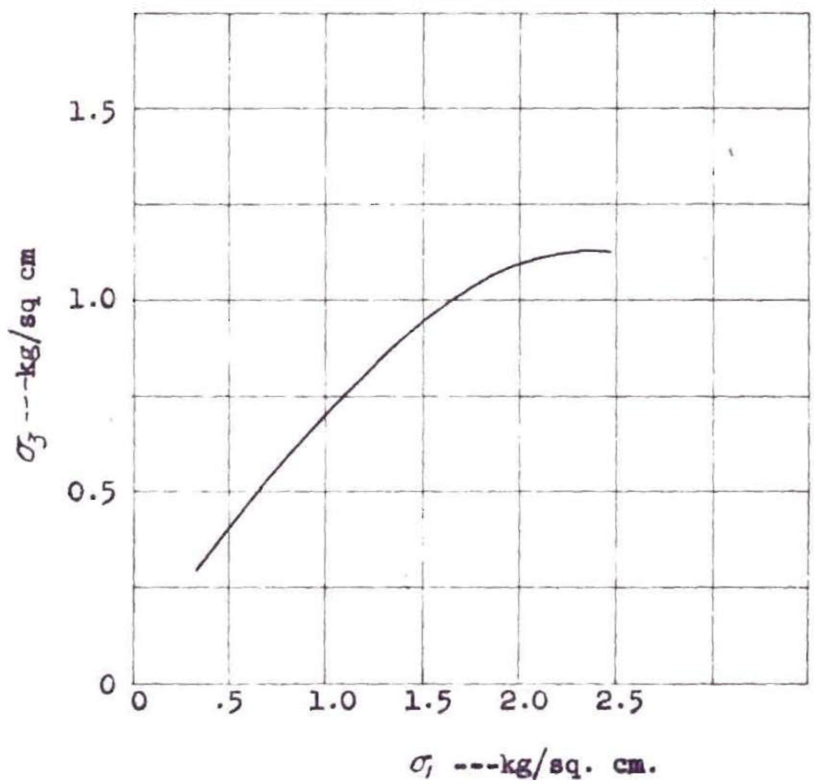
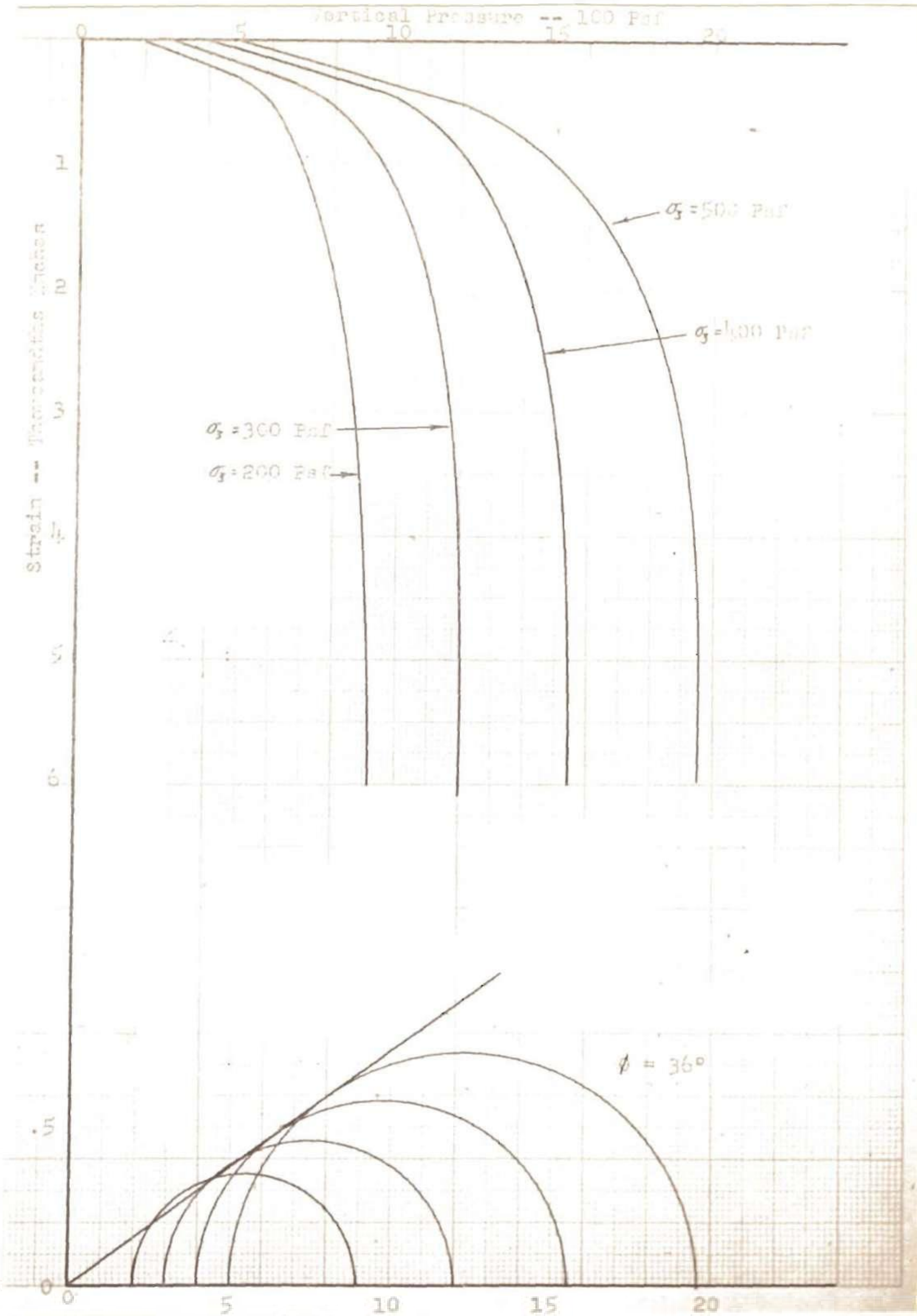


Fig. 18. Variation of Lateral Pressure in Soil Due to Vertical Pressures



Mohr's Circle

Fig. 19. Stress Strain Curves from Triaxial Shear Tests

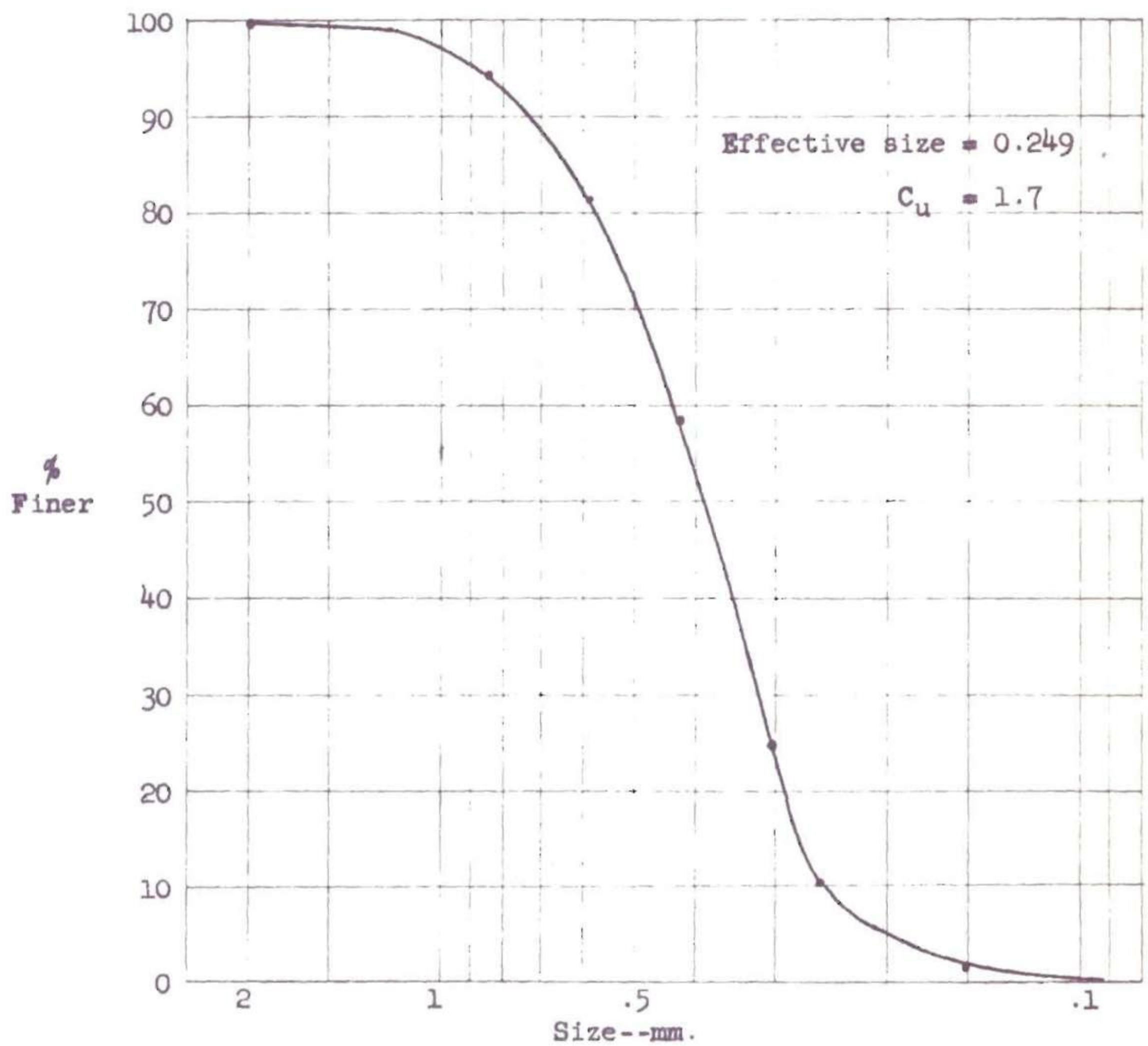


Fig. 20. Grain Size Distribution of Sand

Table 1. Instantaneous Moduli of Elasticity As Scaled From Stress-Strain Curves

	Values of E_1 (Multiply by 10^{+5})			
σ_5 -psf / σ_1 -psf	200	300	400	500
200	0.98			
250	0.98			
300	0.98	1.035		
350	0.98	1.035		
400	0.98	1.035	1.390	
450	0.98	1.035	1.390	
500	0.719	1.035	1.390	1.480
550	0.530	1.035	1.390	
600	0.320	1.035	1.390	1.480
650	0.165		1.390	
700	0.134	0.870	1.390	1.480
750	0.104	0.644	1.390	
800	0.067	0.536	1.390	1.480
850	0.028	0.417	1.390	
900	0.014	0.299		1.480
950	0.244	1.000		
1000		0.189	0.705	1.480
1050		0.125	0.562	
1100		0.083	0.500	1.220
1150		0.056	0.385	
1200		0.032	0.325	0.960
1250			0.256	
1300			0.213	0.757
1350			0.175	
1400			0.132	0.570
1450			0.097	
1500			0.055	0.440
1550				
1600				0.291
1650				
1700				0.217
1750				
1800				0.176

Table 2. Plate Load Test Data
(Zero Corrected)

	Settlement--inches			
Plate / Load-psf	1 sq. ft. Dry	1 sq. ft. Flooded	1.5 sq. ft. Dry	1.5 sq. ft. Flooded
500	.025	.039	.042	.027
1000	.047	.086	.080	.087
1500	.075	.146	.134	.170
2000	.175	.207	.196	.293
2500	.300	----	.272	.412
3000	.530	----	.365	.587
3500	.793	----	.485	.791

Table 3. Plate Load Test Data
(Zero Corrected)

Plate / Load-psf	Settlement --- inches			
	2 sq. ft. Dry	2 sq. ft. Flooded	3 sq. ft. Dry	3 sq. ft. Flooded
500	.052	.073	.071	.078
1000	.104	.149	.128	.152
1500	.150	.232	.192	.241
2000	.210	.330	.260	.335
2500	.275	.428	----	----
3000	.360	.536	.403	.541
3500	.476	.640	----	----
4000	.630	.790	.606	.730
4500	.850	----	----	.994
5000	----	----	.921	----

Table 4. Plate Load Test Data
(Zero Corrected)

Plate / Load-psf	Settlement -- Inches 4 sq ft Dry	4 sq ft Flooded		
500	.079	.073		
1000	.185	.122		
1500	.281	.172		
2000	.379	.214		
3000	.614	.306		
4000	.876	.406		
5000	1.311	.523		
6000	1.886	--		

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